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EXPERIMENTS UPON Z-IRON COLUMNS.

By C. L. STROBEL, M. Am. Soc. C. E.

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About a year ago the writer was called upon to prepare plans and specifications for the superstructure of the new bridge of the Chicago, Milwaukee and St. Paul Railway over the Mississippi River at Kansas City. 1 545 lineal feet of this structure is a double track iron trestle-work of 45 feet average height. It was found that important advantages could be secured for this part of the structure by adopting for the compression members a new form of cross-section consisting of four Z-irons joined by lattice bars in the middle plane, as indicated by sketch.

As Z-irons were not at the time rolled in this country, it was neces sary for the mill furnishing this material to prepare rolls specially for the purpose. The Z-shape is obtained by joining two angles as at a in sketch, and the rolling corresponds to the rolling of angle iron, the rolls operating with their axes parallel to the line $b\,c$. It was found that the

rolling of the Z-iron presented no greater difficulty than the rolling of ordinary angle iron.

Notwithstanding the higher price charged, because new rolls had to be prepared to roll the Z-iron, it was furnished by the mill at a much lower rate per pound than channel iron (for which it acted as a substitute) could have been bought for, and a considerable saving in the cost of the structure was thereby effected.

Other advantages of this form of compression member over two channel bars latticed, the usual type, were thought to be the following:

Economy of material in the lattice bars. These are short, and only one set is required, while two must be used in the other form of cross-section.

Economy of manufacture; only two rows of rivets required, whereas four are needed in the other cross-section.

Protected position of lattice bars and consequent freedom from damage during transportation and in handling.

Possibility of effecting better connections with other members in the design of the structure. This feature was considered an important one, as it is held that the efficiency and serviceability of the parts connected are dependent more upon the method and detail of the connection than upon the particular form of cross-section used for the members themselves.

Having adopted the cross-section described for all the columns and struts in the trestlework of the Kansas City Bridge, it was thought advisable to make some experiments to test its efficiency, and prove that the limits to which the members were to be strained in the structure were safe and consistent.

Owing to the large size of the columns in the structure itself, it was not possible to test full-size specimens, therefore smaller columns were used in the experiments, but of the proportions, as near as possible, of those in the structure. The Z-irons in the latter were 3 x 5 x 3 inches in size and $\frac{1}{16}$ inches thick, while in the test columns $2\frac{1}{2}$ x 3 x $2\frac{1}{2}$-inch Z-irons were used-$\frac{5}{16}$-inches thick. Six columns were tested of the lengths 10 feet $11\frac{1}{4}$ inches, 15 feet, and 19 feet $\frac{3}{4}$ inches, two columns for each length. The expense of these tests were borne by the Chicago,

Milwaukee and St. Paul Railway Company. The results proved to be very satisfactory, and the subject was considered of sufficient general importance to be pursued further. It was decided to test nine additional columns of longer lengths, so as to obtain results covering the limits of actual practice. The expense of these tests was jointly borne by the Chicago, Milwaukee and St. Paul Railway Company, and the Keystone Bridge Company. The additional columns tested were 22, 25 and 28 feet long, three columns for each length.

The results of the tests of the entire series of fifteen columns are herewith presented to the Society as a contribution on the subject of the strength of wrought-iron compression members.

The tests were made in the hydraulic testing machine of the Keystone Bridge Company, at their works in Pittsburgh, under the supervision of S. C. Weiskopf, M. Am. Soc. C. E., inspector of shop work, on behalf of the Chicago, Milwaukee and St. Paul Railway Company, for the Kansas City Bridge.

The material was manufactured by Brown, Bonnell & Co., of Youngstown, O. The tests made on specimens cut from the Z-irons are shown in Table No. 3. These tests comprise all that were made on the Z-iron of this size furnished for the Kansas City Bridge. It is difficult to determine the elastic limit accurately on the testing machine used (Riehle's 100 000 pounds machine), and it is probable that the values obtained are too high by several thousand pounds to the square inch in each case.

The first six columns tested were made by M. Lassig, M. Am. Soc. C. E., of Chicago, and the last nine columns by the Keystone Bridge Company, of Pittsburgh.

The columns were placed horizontally in the testing machine, with the lattice bars in a vertical plane, the ends of the columns abutting squarely against the castings of the testing machine without the interposition of shoes. To counteract the tendency of the column to sag from its own weight, an upward pull was caused to be exerted against the middle of the column equal to one-half its own weight.

Plates XXXI and XXXII show the columns tested in detail. Their appearance after the test is plainly shown by the photographs, Plates XXXI, XXXII, XXXIII and XXXIV.

All of the columns failed in the direction in which they were theoretically weakest, and in a uniform manner by deflection.

The results, both in general tabulated form and in detail, will be found appended, Tables Nos. 1 and 2, also a diagram (Plate XXXV), showing the ultimate strengths obtained for all square ended columns tested up to date. This diagram is a reproduction in part of Plate XXXV in Mr. Jos. M. Wilson's paper on "Specifications for Strength of Iron Bridges," Transactions, Vol. XV, p. 366, June, 1886. The tests on octagon columns with diametrical bolts are not included in this diagram because the writer considers this column faulty in design, and it never obtained an extended use and is obsolete now.

It will be seen that the Z-iron columns compare favorably with other columns in ultimate resistance. The values obtained are near approximations to the Watertown results with Phœnix columns, and exceed those heretofore obtained with other types of columns. This favorable showing for the Z-iron columns should probably be attributed to the fact that the material in the outer periphery of the cross-section, on which dependence must be placed to hold the column in line, is not weakened by rivet holes, but is left solid and unbroken, and is, therefore, in the best shape to do its work effectively.

The writer called attention to the small variation in the ultimate strength of columns of the same type for lengths not exceeding 90 radii of gyration (32 diameters), in his discussion of "Experiments upon Phœnix Column" (Transactions, Vol. XI, p. 98, February, 1882). Excluding very short lengths it was found that the ultimate strength up to at least the limit mentioned (90 radii), was practically constant irrespective of the length, though not the same for different types of columns.

The average for Hodgkinson's cylindrical tubes, for lengths ranging from 40 to 90 radii (14 to 32 diameters) was 33 100 pounds per square inch.

The average for the Watertown tests of Phœnix columns for lengths ranging from 40 to 110 radii (14.1 to 39.5 diameters) was 35 800 pounds per square inch.

The average for Mr. Bouscaren's tests of latticed channel columns for lengths ranging from 59 to 90 radii (21 to 32 diameters) was 32 300 pounds per square inch.

The diameters referred to in the above are the diameters of an equivalent cylindrical tube whose area is concentrated in the circumference and whose radius of gyration is the same as that of the section considered.

The present tests on Z-iron columns show no variation in the ultimate strengths for columns 64 radii (23 diameters) and 88 radii (31 diameters) long, the average being 35 650 pounds per square inch.

Tests made by the Keystone Bridge Company for the Baltimore and Ohio Railroad during the construction of the Susquehanna River Bridge, near Port Deposit, on pin bearing steel columns ranging in length from 66 to 98 radii, show a striking uniformity in the ultimate strength.

The only tests which do not seem to conform to the rule mentioned are Mr. Christie's experiments on angles, tees and channels; but these tests are very irregular in their results and seem to defy all law. An explanation for this may, perhaps, be found in the different conditions of end bearing from those which obtain in columns. In the latter the center of gravity is in the center of the section so that the forces acting at the ends of the column have considerable leverage and are disposed to best advantage to hold the column in line. In solid sections like angles, tees and channels, on which Mr. Christie's experiments were made, not only in the metal grouped nearer to the center of gravity and the end forces therefore act with a smaller leverage, but, owing to the want of symmetry of these sections, there is much less resistance to deflection of the strut in one direction than another.

Considering all the tests which have been made on hollow columns with square ends, the writer believes that he is justified in assuming that their ultimate strength is constant for lengths up to 90 radii (32 diameters). This assumption is a convenience in the practical work of proportioning the parts of a bridge, because it saves the necessity of calculating the radius of gyration for the chords and end posts, which is a somewhat laborious operation and cannot readily be simplified by means of tables.

In devising a formula which would satisfy the results of the tests on Z-iron columns for lengths beyond 90 radii, the equation of a straight line was found to answer as well as the equation of a curve, while having the advantage of simplicity. The following is the formula selected:

Ultimate strength in pounds per square inch $= 46\,000 - 125\,\frac{l}{r}$ for lengths exceeding 90 radii and 35 000 for lengths equal or less than 90. The diagram, Plate XXXV, gives a graphic representation of the formula, values for which are also contained in the Table No. 1. The deviations from the results of the tests are on the safe side.

It has been customary to allow 8 000 pounds per square inch in compression for bridge members of short length, which corresponds to a factor of safety of $\frac{35000}{8000} = 4.35$, when taken with reference to the ultimate strength. Although this factor is slightly less than is customary for tension members (for which the factor is about $\frac{46000}{10000} = 4.6$), it seems ample when the fact is considered that the ratio of elastic limit to allowed strain is the true factor of safety in the ordinary use of a structure, and that this ratio will be greater for compression members than for tension members on the basis of above factor. Indeed, for long compression members which fail by flexure as a whole, the ultimate strength is very little removed from the elastic limit, so that the factor of safety with reference to the elastic limit becomes greater as the length of the member increases, assuming the factor of safety with reference to the ultimate or crippling strength to remain constant. Dividing the constants in the above formula by 4.35, we obtain $10\,600 - 28.5\,\frac{l}{r}$.

For convenience, and as providing additional security for long members, the writer considers it desirable to substitute 30 for 28.5 as the second constant, whence is obtained the following:

Allowed strain in pounds per square inch for square ended Z-iron columns,

$$10\,600 - 30\,\frac{l}{r} \text{ for lengths exceeding 90 radii;}$$

and 8 000 for lengths equal or less than 90 radii.

TABLE No. 1.

General Results of Compression Tests of 15 Z-Iron Columns.

Radius of gyration, lattice bars not considered, = 2.05.

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No. of Statement	No of Column.	Length of Column.	Weight, Pounds.	Sectional Area. Square inches.	Ultimate Strength. Actual.		Ratio of length to least radius of gyration.	Ult. Str by formula (Rankine Gordon) $\frac{36\,000}{1+\frac{l^2}{36\,000\,r^2}}$	Ult. Str. by formula $46\,000-125\frac{l}{r}$
					Total Pounds.	Pounds per sq. in.			
		Feet. In.							
1	3	10 $11\frac{1}{4}$	421	9 435	347 100	36 800	64	32 300	
2	4	"	441	9 984	345 800	34 600	"	"	
3	2	15 0	570	9 480	328 000	34 600	88	29 600	35 000
4	5	"	560	9 280	340 000	36 600	"	"	"
5	1	19 $\frac{3}{4}$	699	9 241	312 000	33 800	112	26 700	32 200
6	6	"	754	10 104	340 600	33 700	"	"	"
7	7	22 0	805	9 286	285 000	30 700	129	24 600	29 900
8	8	"	805	9 286	274 300	29 500	"	"	"
9	9	"	805	9 286	285 000	30 700	"	"	"
10	10	25 0	900	9 156	257 400	28 100	146	22 600	27 750
11	11	"	925	9 456	265 200	28 000	"	"	"
12	12	"	930	9 516	270 400	28 400	"	"	"
13	13	28 0	1 025	9 375	260 000	27 700	164	20 600	25 500
14	14	"	1 050	9 643	270 000	28 000	"	"	"
15	15	"	1 025	9 375	258 700	27 600	"	"	"

TABLE No. 2.

STATEMENT IN DETAIL OF TESTS.

Statement No. 1. Column No. 3.

Length of column.....10 feet 11¼ inches.
Sectional area, 9.435 square inches.
Weight.....427 pounds.
Contraction measured in.....9 feet.
Plane of lacing of column placed vertical.

Load.	Vert. Deflection.		Hor. Deflection.		Contraction.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
40 000	....	....	.04	....	.02	....
250 000	.04	....	.10	....	.10	....
270 000	.04	....	.11	.02	.11	....
280 000	.05	.03	.15	.02	.12	....
290 000	.05	.03	.17	.02	.12	....
300 000	.10	.04	.15	.07	.13	.01
347 100	Ultimate	strength.	Failed hor	izontally.		

Statement No. 2. Column No. 4.

Length of column.....10 feet 11¼ inches.
Sectional area, 9.984 square inches.
Weight.....427 pounds.
Contraction measured in.....9 feet.
Plane of lacing of column placed vertical.

Load.	Vert. Deflection.		Hor. Deflection.		Contraction.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
40 000	....	....	....	....	....	....
100 000	....	....	.03	....	.04	....
180 000	.03	....	.03	....	.07	....
200 000	.03	....	.05	....	.07	....
220 000	.03	....	.05	....	.09	....
240 000	.03	....	.07	....	.10	....
260 000	.04	....	.10	....	.10	....
280 000	.04	....	.12	....	.12	.02
300 000	.04	....	.10	.03	.14	.03
310 000	.04	....	.10	.04	.14	.03
320 000	.04	....	.12	.04	.14	.04
330 000	.05	.05	.12	.04	.14	.04
340 000	.15	.10	.12	.07	.15	.06
345 800	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 3. Column No. 2.

Length of column ..... 15 feet.
Sectional area, 9.480 square inches.
Weight ..... 565 pounds.
Contraction measured in ..... 13 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
40 000	.06	....	.02	....	.02	....
200 000	.06	....	.12	....	.11	....
220 000	.06	....	.10	....	.12	....
240 000	.06	....	.09	....	.14	....
260 000	.06	....	.12	....	.15	....
280 000	.06	....	.13	.02	.15	....
290 000	.06	....	.13	.03	.16	....
300 000	.06	....	.19	.04	.18	....
310 000	.06	....	.19	.07	.19	.01
320 000	.06	....	.28	.13	.20	.04
328 000	Ultimate	strength.	Failed hor	izontally.		

Statement No. 4. Column No. 5.

Length of column ..... 15 feet.
Sectional area, 9.280 square inches.
Weight ..... 565 pounds
Contraction measured in ..... 13 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
40 000	.12	....	.01	....	.03	....
100 000	.12	....	.01	....	.05	....
150 000	.12	....	....	.. .	.08	....
180 000	.12	....	.03	....	.09	. ..
200 000	.16	....	.03	....	.10	....
220 000	.17	.02	.03	....	.12	....
240 000	.17	.02	.03	....	.14	....
260 000	.22	.02	.07	....	.15	.01
280 000	.24	.06	.07	.02	.15	.02
290 000	.26	.09	.07	.04	.16	.02
300 000	.27	.11	.10	.04	.17	.03
310 000	.27	.11	.11	.05	.18	.04
320 000	.27	.12	.11	.09	.19	.04
330 000	.39	.17	.14	.10	.19	.04
340 000	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 5. Column No. 1.

Length of column..... 19 feet $\frac{3}{4}$ inches.
Sectional area, 9.241 square inches.
Weight .....729 pounds.
Contraction measured in ..... 17 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
40 000	.05	....	.15	....	....	....
50 000	.05	....	.15	....	.01	....
60 000	.07	....	.15	....	.02	....
70 000	.07	....	.15	....	.03	....
80 000	.07	....	.15	....	.05	....
90 000	.09	....	.15	....	.05	....
100 000	.09	....	.15	....	.06	....
110 000	.09	....	.15	....	.06	....
120 000	.09	....	.15	....	.07	....
130 000	.09	....	.15	....	.07	....
140 000	.11	....	.15	....	.08	....
150 000	.11	....	.15	....	.09	....
160 000	.11	....	.17	....	.10	....
170 000	.11	....	.17	....	.11	....
180 000	.12	....	.17	....	.12	....
190 000	.14	....	.17	....	.12	....
200 000	.14	....	.17	....	.13	....
210 000	.14	....	.17	....	.13	....
220 000	.14	.05	.17	....	.14	....
230 000	.17	.05	.17	....	.15	....
240 000	.17	.05	.17	....	.16	....
250 000	.18	.07	.17	....	.17	....
260 000	.20	.08	.17	.03	.18	....
270 000	.22	.09	.14	.03	.19	....
280 000	.27	.09	.14	.03	.20	.02
312 000	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 6. Column No. 6.

Length..... 19 feet $\frac{3}{4}$ inches.
Sectional area, 10.104 square inches.
Weight..... 729 pounds.
Contraction measured in..... 17 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
80 000	.08	....	.14	....	.04	....
100 000	.07	....	.14	....	.05	....
150 000	.07	....	.14	....	.08	....
180 000	.08	....	.14	....	.11	....
200 000	.08	....	.14	....	.12	....
220 000	.08	....	.12	....	.14	....
240 000	.10	.02	.12	.02	.16	....
260 000	.10	.02	.12	.03	.17	....
280 000	.11	.04	.12	.04	.19	.01
300 000	.11	.02	.08	.04	.21	.02
310 000	.12	.04	.10	.05	.22	.02
320 000	.13	.04	.10	.07	.23	.03
330 000	.13	.04	.10	.07	.24	.03
340 000	.11	.05	.17	.15	.24	.06
340 600	Ultimate strength.		Failed horizontally.			

STATEMENT IN DETAIL OF TESTS.

Statement No. 7. Column No. 7.

Length of column..... 22 feet.
Sectional area, 9.286 square inches.
Weight..... 805 pounds.
Contraction measured in..... 20 feet.
Plane of lacing of column placed vertical.

Load.	Vert. Deflection.		Hor. Deflection.		Contraction.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.07	....	.31	.02	.07	....
150 000	.08	....	.33	.04	.12	....
170 000	.08	....	.37	.05	.15	....
190 000	.08	....	.37	.05	.16	....
200 000	.08	.02	.38	.07	.16	....
210 000	.08	.02	.38	.07	.16	....
220 000	.08	.03	.42	.11	.19	....
230 000	.08	.03	.42	.11	.19	....
240 000	.08	.03	.43	.12	.23	....
250 000	.08	.03	.46	.13	.23	.01
260 000	.08	.03	.49	.17	.24	.01
270 000	.08	.03	.69	.31	.26	.03
280 000	.08	.03	1.04	.58	.28	.05
285 000	Ultimate	strength.	Failed hor	izontally.		

Statement No. 8. Column No. 8.

Length of column..... 22 feet.
Sectional area, 9.286 square inches.
Weight..... 805 pounds.
Contraction measured in..... 20 feet.
Plane of lacing of column placed vertical.

Load.	Vert. Deflection.		Hor. Deflection.		Contraction.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	....	....	.30	....	.12	....
150 000	....	....	.30	....	.12	....
170 000	....	....	.35	....	.14	....
190 000	....	....	.36	....	.14	....
200 000	....	....	.38	....	.17	....
210 000	....	....	.38	....	.18	....
220 000	....	....	.40	....	.18	....
230 000	....	....	.40	.04	.20	.01
240 000	.03	....	.46	.08	.22	.01
250 000	.04	.01	.52	.11	.22	.01
260 000	.04	.02	.66	.17	.23	.02
270 000	.07	.03	1.00	.48	.27	.04
274 300	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 9. Column No. 9.

Length of column..... 22 feet.
Sectional area, 9.286 square inches.
Weight..... 805 pounds.
Contraction measured in..... 20 feet.
Plane of lacing of column placed vertical.

Load.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.08	....	.31	....	.07	....
150 000	.08	....	.31	....	.11	....
170 000	.09	.01	.33	.01	.12	....
190 000	.09	.01	.34	.02	.14	....
200 000	.09	.01	.34	.02	.14	....
210 000	.10	.01	.35	.04	.16	....
220 000	.10	.01	.37	.07	.17	....
230 000	.10	.01	.37	.06	.18	.01
240 000	.10	.01	.38	.07	.18	.01
250 000	.10	.01	.40	.09	.22	.02
260 000	.10	.01	.43	.12	.22	.02
270 000	.08	.01	.54	.20	.25	.02
280 000	.08	.01	.77	.35	.27	.04
285 000	Ultimate	strength.	Failed hor	izontally.		

Statement No. 10. Column No. 10.

Length of column..... 25 feet.
Sectional area, 9.156 square inches.
Weight..... 900 pounds.
Contraction measured in..... 23 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.04	....	.31	....	.09	....
150 000	.04	.02	.38	.03	.16	....
170 000	.04	.02	.40	.04	.17	....
190 000	.04	.02	.43	.07	.19	....
200 000	.05	.03	.47	.09	.20	....
210 000	.05	.03	.52	.10	.21	....
220 000	.06	.03	.58	.13	.22	....
230 000	.06	.03	.64	.16	.24	....
240 000	.07	.04	.77	.22	.25	....
250 000	.10	.04	1 03	.35	.26	....
257 400	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 11. *Column No.* 11.

Length of column..... 25 feet.
Sectional area, 9.456 square inches.
Weight..... 925 pounds.
Contraction measured in..... 23 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.05	....	.08	....	.08	....
150 000	.05	....	.13	....	.13	....
170 000	.05	....	.17	.02	.17	....
190 000	.05	....	.22	.03	.17	....
200 000	.05	....	.26	.05	.19	....
210 000	.05	....	.28	.06	.20	.01
220 000	.05	....	.36	.07	.22	.01
230 000	.05	.02	.41	.09	.23	.02
240 000	.02	.04	.47	.12	.23	.02
250 000	.02	.04	.62	.16	.25	.02
260 000	.05	.03	.98	.26	.28	.02
265 200	Ultimate	strength.	Failed hor	izontally.		

Statement No 12. *Column No.* 12.

Length of column..... 25 feet.
Sectional area, 9.516 square inches.
Weight..... 930 pounds.
Contraction measured in..... 23 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.11	....	.26	....	.06	....
150 000	.11	....	.31	.02	.12	....
170 000	.11	....	.33	.03	.13	....
190 000	.11	....	.36	.03	.15	....
200 000	.11	....	.39	.04	.19	....
210 000	.11	....	.41	.05	.19	....
220 000	.08	....	.45	.06	.19	....
230 000	.08	.02	.48	.07	.22	....
240 000	.08	.02	.57	.09	.24	....
250 000	.08	.02	.66	.14	.25	....
260 000	.08	.02	.84	.23	.26	....
270 000	.08	.02	1.30	.50	.27	.02
270 400	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 13. Column No. 13.

Length of column ..... 28 feet.
Sectional area, 9.375 square inches.
Weight ..... 1 025 pounds.
Contraction measured in ..... 26 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.18	...	.18	....	.08	....
150 000	.18	....	.25	....	.16	....
170 000	.18	....	.25	....	.17	....
190 000	.18	.01	.25	....	.20	.01
200 000	.18	.01	.27	....	.22	.01
210 000	.18	.02	.32	....	.23	.01
220 000	.14	.02	.32	....	.25	.01
230 000	.13	.01	.40	.04	.26	.01
240 000	.13	....	.50	.06	.27	02
250 000	.11	.01	.67	.12	.27	.02
260 000	Ultimate	strength.	Failed hor	izontally.		

Statement No. 14. Column No. 14.

Length of column ..... 28 feet.
Sectional area, 9.643 square inches.
Weight ..... 1 050 pounds.
Contraction measured in ..... 26 feet.
Plane of lacing of column placed vertical.

LOAD.	VERT. DEFLECTION.		HOR. DEFLECTION.		CONTRACTION.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.11	....	.26	....	.12	....
150 000	.11	.01	.26	....	.16	....
170 000	.13	.93	.31	.02	.17	....
190 000	.15	.05	.33	.03	.21	....
200 000	.15	.05	.34	.05	.23	....
210 000	.16	.06	.34	.05	.24	....
220 000	.18	.07	.36	.07	.25	....
230 000	.18	.07	.41	.09	.25	....
240 000	.21	.08	.47	.11	.26	....
250 000	.24	.09	.55	.13	.27	.01
260 000	.25	.11	.67	.17	.27	.01
270 000	Ultimate	strength.	Failed hor	izontally.		

STATEMENT IN DETAIL OF TESTS.

Statement No. 15. *Column No.* 15.

Length of column..... 28 feet.
Sectional area, 9.375 square inches.
Weight..... 1 025 pounds.
Contraction measured in..... 26 feet.
Plane of lacing of column placed vertical.

Load.	Vert. Deflection.		Hor. Deflection.		Contraction.	
	Temp.	Perm.	Temp.	Perm.	Temp.	Perm.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
100 000	.13	....	.10	....	.10	....
150 000	.07	....	.18	....	.14	....
170 000	.10	.01	.23	.01	.17	....
190 000	.11	.02	.28	.02	.21	....
200 000	.13	.02	.32	.03	.23	....
210 000	.13	.03	.37	.04	.23	....
220 000	.14	.03	.41	.05	.23	....
230 000	.15	.03	.48	.07	.24	....
240 000	.18	.04	.62	.07	.25	....
250 000	.20	.05	.80	.18	.26	....
258 700	Ultimate strength.		Failed horizontally.			

DISCUSSION ON THE STROBEL COLUMN.

By D. J. Whittemore, Past President Am. Soc. C. E.

When I learned that Mr. Strobel would offer a paper for our Transactions on this column, I was exceedingly anxious to secure an advance proof copy of it, as on the result of the tests depended in some considerable measure the safety of an important structure built under my chief direction. On the perusal of that proof I find that Mr. Strobel has fully covered those points to which I wished to call attention, hence I have but little to offer.

When Mr. Strobel first suggested the use of this form of column, I am free to say it did not strike me with favor, but giving it further thought, I concluded to permit its use in the structure under my charge, and it is exceedingly gratifying to me, that the record of tests fully corroborated Mr. Strobel's predictions, given before the tests were made. We must admit, however, that Mr. Strobel was not without some authority in making this prediction, as by referring back to his remarks on the tests of the Phœnix columns, published in the Transactions some years ago, we find that he then brought to our attention a form quite similar and the resulting test of it. When we consider that this mass of metal is brought to the requisite form by driving two rows of rivets, its economic cost of manufacture becomes apparent. In this respect it differs and has no advantage over what we designate as the American column, which it resembles to some extent. I wish to acknowledge one fact that operated to govern me somewhat in consenting to the use of the column, and that was that all the parts were outside of the prevaling pool arrangement between makers of channels and beams, of which a higher price must be paid than for angles and other forms. This in our work amounted to several thousand dollars, so we find ourselves many times repaid for whatever was expended for tests.

It strikes me as quite proper that this paper should first appear in our Transactions, since the design of the column is from one of our accomplished members, the consent of its use in an important structure was given by a member; it was manufactured and it was tested by members of our Society. It strikes me that the name given it, "Z-iron Column," is not a proper one. The forms that go to make this column are now being made of steel, and undoubtedly steel columns will be made of these forms; hence I suggest the propriety of designating this hereafter as the Strobel column, after its designer, and let him to that extent at least have whatever credit shall inure to its design.

TABLE No. 3.

Tests of Specimens cut from 3-inch Z-Iron.

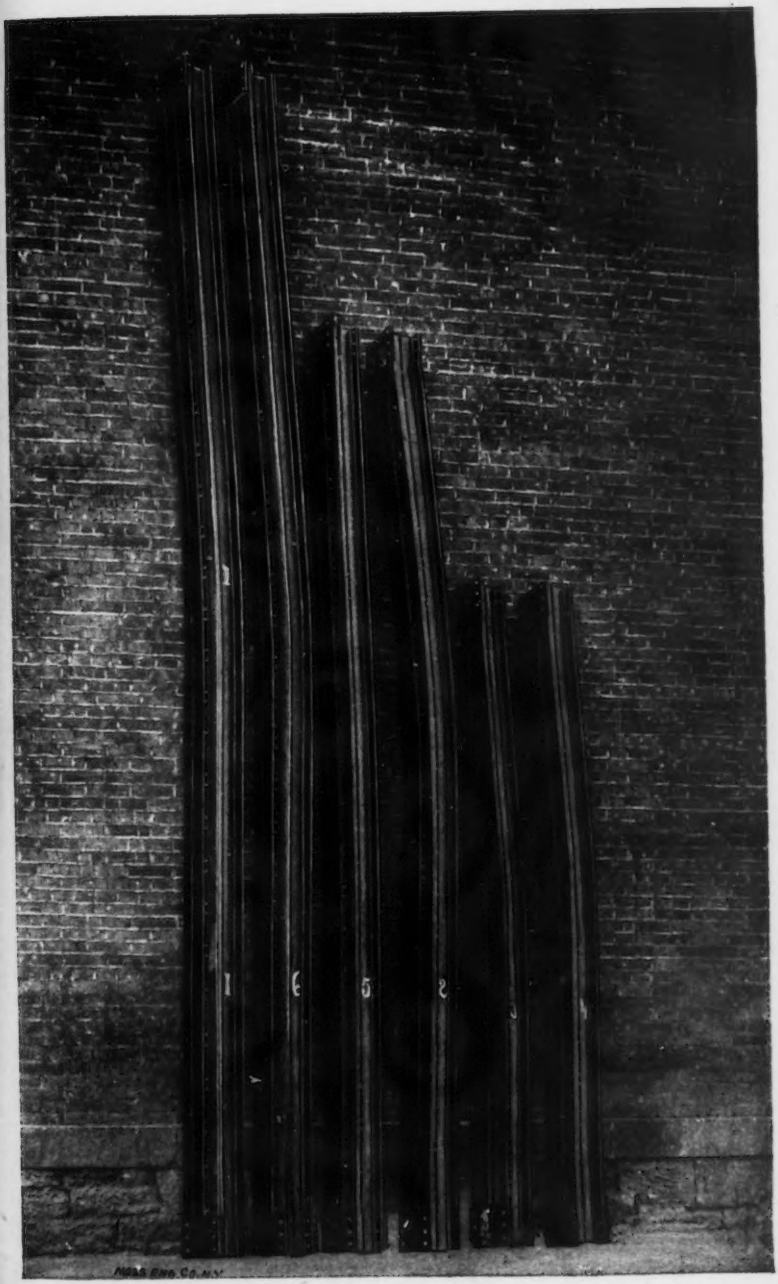
(Z-irons $2\frac{1}{2} \times 3 \times 2\frac{1}{2} \times \frac{5}{16}$ inches, 8 pounds per foot.)

Iron Manufactured by Brown, Bonnell & Co., Youngstown, Ohio.—Tests made in 100 000 pounds Riehle Machine, at Youngstown, Ohio.

Vol. XVIII, p. 118.

Date.	Number of Test.	Test Piece Cut From.	Cross Dimensions.		Area.			Elastic Limit.		Maximum Load.		Extensions.		Accepted or Rejected.	Remarks.	Description of Fracture.
			Original.	Fractured.	Original.	Fractured.	Per Cent. of Reduction.	Actual.	Lbs. per Sq. In.	Actual.	Lbs. per Sq. In.	In 8 Inches.	Per Cent. in 8 Inches.			
1887.		Inches.														
Feb. 16.	1	$2\frac{1}{2} \times 3 \times 2\frac{1}{2}$ Z 8	1.433 × .329	1.357 × .285	.4710	.3920	16.70	16 130	34 200	24 100	51 160	1.20	15.00	Accepted.	Web.	Fracture, fibrous, clean and compact.
"	2	"	1.444 × .329	1.325 × .285	.4750	.3760	20.80	16 230	34 170	24 630	51 850	1.48	18.50	"	Flange.	" "
"	3	"	1.449 × .329	1.365 × .280	.4870	.3820	21.60	17 280	35 480	25 080	51 500	1.04	13.00	"	Web.	flaw in specimen.
"	4	"	1.454 × .333	1.350 × .280	.4840	.3780	21.90	16 940	35 000	25 600	52 900	1.54	19.30	"	Flange.	clean and compact.
"	5	"	1.481 × .331	1.410 × .290	.4930	.4090	17.10	17 280	35 000	24 440	49 570	0.96	12.00	"	Web.	showing crop end.
"	6	"	1.498 × .326	1.392 × .284	.4880	.3950	19.00	17 260	35 300	25 200	51 700	1.32	16.50	"	Flange.	clean and compact.
"	7	"	1.485 × .332	1.394 × .290	.4930	.4040	18.10	16 480	33 400	24 720	50 150	1.12	14.00	"	Web.	"
"	8	"	1.498 × .326	1.345 × .275	.4880	.3700	24.20	16 600	34 000	25 320	51 880	1.58	19.70	"	Flange.	"
" 17.	9	"	1.398 × .327	1.284 × .267	.4560	.3430	24.80	14 960	32 830	23 120	50 700	1.48	18.50	"	"	"
"	10	"	1.404 × .330	1.322 × .288	.4630	.3800	17.50	15 180	32 760	22 700	49 000	1.13	14.00	"	Web.	showing crop end.
"	11	"	1.399 × .333	1.265 × .270	.4660	.3520	24.40	16 430	35 360	24 530	52 650	1.81	22.70	"	Flange.	clean and compact.
"	12	"	1.500 × .326	1.416 × .276	.4890	.3910	20.00	16 850	34 460	23 880	48 800	0.90	11.30	"	Web.	shows cinder flaw.
"	13	"	1.492 × .325	1.410 × .280	.4850	.3950	18.60	16 870	34 790	24 780	51 100	1.20	15.00	"	Flange.	clean and compact.
" 18.	14	"	1.492 × .330	1.407 × .384	.4920	.4000	18.70	16 970	34 490	24 740	50 300	1.13	14.10	"	Web.	broke in grips.
"	15	"	1.498 × .335	1.370 × .280	.5020	.3840	23.10	17 300	34 450	25 470	50 730	1.72	21.50	"	Flange.	clean and compact.
"	16	"	1.502 × .326	1.438 × .290	.4960	.4170	15.00	16 870	34 400	24 950	50 900	1.12	14.00	"	Web.	broke in grips.
"	17	"	1.497 × .333	1.384 × .280	.4990	.3870	22.50	17 140	34 100	25 140	50 400	1.30	16.50	"	Flange.	clean and compact.
"	18	"	1.491 × .331	1.376 × .275	.4940	.3780	23.50	16 870	34 180	25 260	51 180	1.61	20.10	"	"	" "
"	19	"	1.455 × .355	1.317 × .261	.4870	.3440	29.50	15 210	31 230	23 900	49 060	1.54	19.25	"	"	" "
"	20	"	1.480 × .337	1.380 × .281	.4990	.3880	22.20	15 960	32 000	24 520	49 120	1.28	16.00	"	Web.	" "
"	21	"	1.370 × .324	1.262 × .274	.4440	.3560	20.60	15 340	34 300	23 680	53 300	1.40	17.50	"	"	" "
"	22	"	1.354 × .330	1.245 × .279	.4470	.3470	22.30	14 570	32 600	23 250	52 000	1.48	18.50	"	"	" "
April 9.	23	3 × $2\frac{1}{2} \times \frac{5}{16}$ Z	1.392 × .330	1.293 × .265	.4594	.3427	25.00	15 420	33 570	21 650	47 130	0.85	11.00	Rejected.	Flange.	" short and bad.
"	24	"	1.383 × .333	1.258 × .273	.4605	.3434	25.00	16 750	36 380	24 380	52 940	1.66	20.75	Accepted.	"	" clean and compact.
"	25	"	1.370 × .334	1.244 × .277	.4576	.3446	25.00	14 970	32 720	23 240	50 790	1.66	20.75	"	"	" "
"	26	"	1.456 × .321	1.353 × .263	.4674	.3558	24.00	16 780	35 900	25 630	54 820	1.52	19.00	"	Web.	" "
"	27	"	1.460 × .322	1.361 × .263	.4701	.3580	24.00	16 400	34 890	24 770	52 700	1.26	16.00	"	"	" "
" 13.	28	"	1.483 × .320	1.367 × .266	.4746	.3636	23.00	15 800	33 290	23 240	48 970	0.89	11.00	Rejected.	"	Fracture, unwelded, dirty.
"	29	"	1.490 × .340	1.340 × .280	.5066	.3752	26.00	18 240	36 010	26 250	51 820	1.59	20.00	Accepted.	Flange.	Fracture, fibrous, clean and compact.
"	30	"	1.430 × .337	1.332 × .280	.4819	.3729	23.00	16 340	33 910	23 920	49 640	0.96	12.00	"	"	" "
"	31	"	1.421 × .322	1.335 × .275	.4576	.3671	20.00	15 450	33 770	22 440	49 040	0.89	12.00	"	Web.	" "
May 11.	32	"	1.373 × .349	1.251 × .296	.4792	.3703	23.00	16 700	34 850	24 720	51 920	1.44	18.00	"	Flange.	" "
"	33	"	1.400 × .334	1.237 × .275	.4676	.3402	27.00	15 750	33 680	23 730	50 750	1.61	20.00	"	"	" "
"	34	"	1.369 × .329	1.238 × .265	.4504	.3281	27.00	15 930	35 370	23 220	51 550	1.56	20.00	"	"	" "
"	35	"	1.376 × .315	1.275 × .262	.4334	.3341	23.00	14 160	32 670	20 040	46 240	0.91	11.00	Rejected.	Web.	" short, bad flaw.
"	36	"	1.375 × .324	1.297 × .283	.4455	.3670	18.00	15 260	34 250	21 620	49 000	1.00	12.50	Accepted.	"	" clean and compact.
"	37	"	1.346 × .319	1.262 × .272	.4294	.3433	20.00	14 680	34 190	20 910	49 000	1.04	13.00	"	"	" "
"	38	"	1.458 × .315	1.359 × .264	.4593	.3588	23.00	15 070	32 810	22 720	49 470	1.25	16.00	"	"	" "

Note.—Bending tests and nicking tests made, one each for every tension test, and they were found very satisfactory. Brackets to right of sheet connecting any two tests, as for instance tests Nos. 19 and 20, indicate that these specimens were cut from same piece of Z, one from the flange and one from the stem or web.


1
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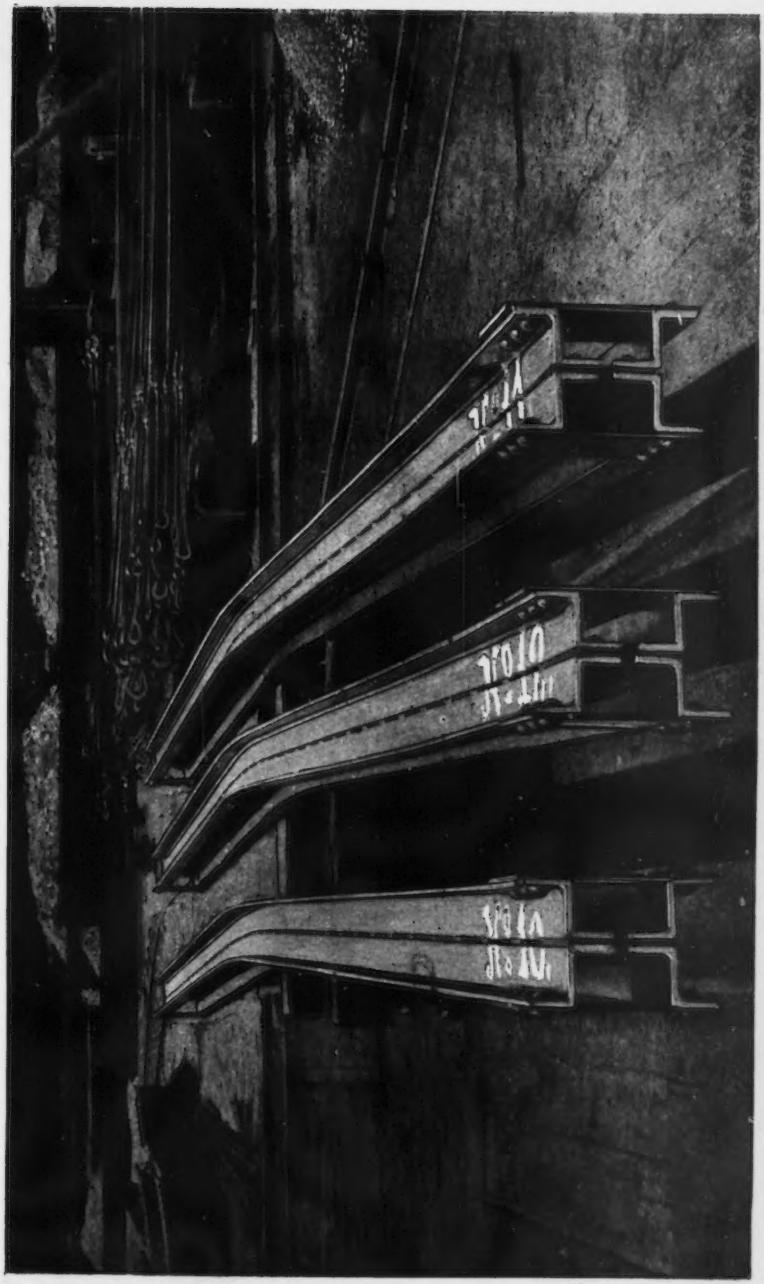
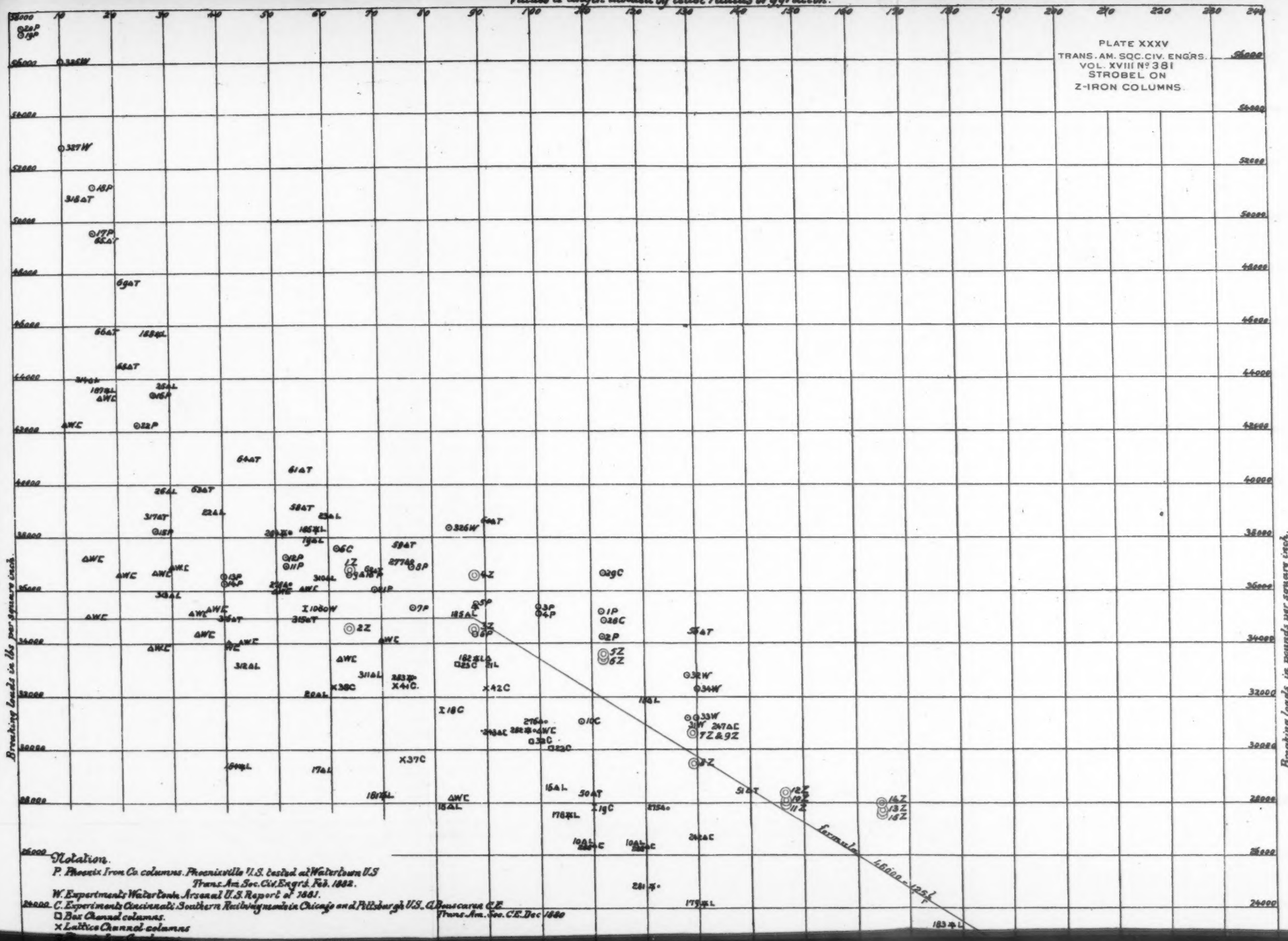
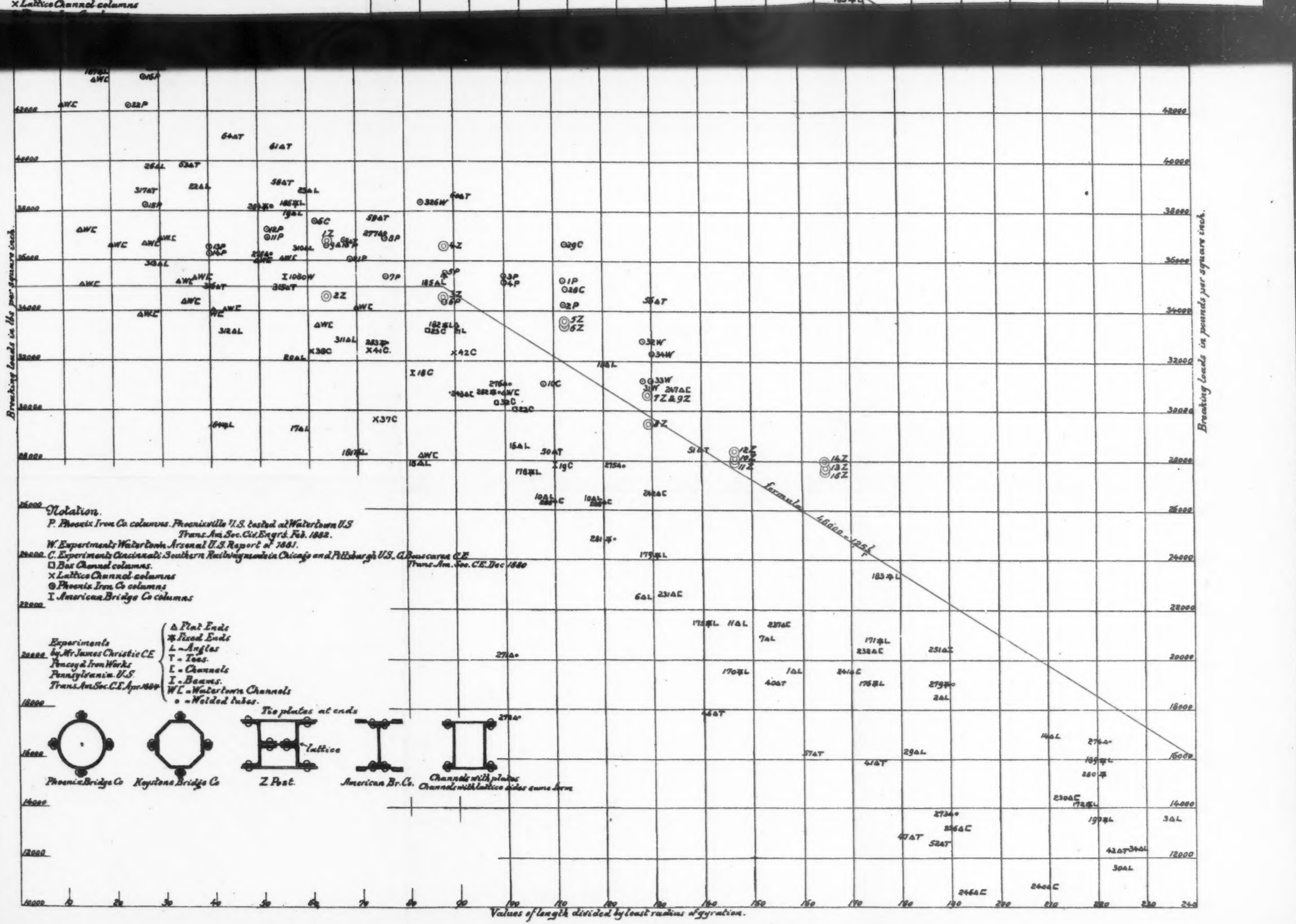
A
MOSS ENG. CO. N.Y.

PLATE XXXII.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XVIII. NO. 381.
STROBEL ON
Z-IRON COLUMNS.




Values of length divided by least radius of gyration.
Breaking loads in lbs per square inch.
Breaking loads in pounds per square inch.
PLATE XXXV
TRANS. AM. SOC. CIV. ENGRS.
VOL. XVIII Nº 381
STROBEL ON
Z-IRON COLUMNS.
formula 48000 − 125 l
Notation.
P. Phoenix Iron Co columns. Phoenixville U.S. tested at Watertown U.S. Trans. Am. Soc. Civ. Engrs. Feb. 1882.
W. Experiments Watertown Arsenal U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway made in Chicago and Pittsburgh U.S. A. Bouscaren C.E. Trans. Am. Soc. C.E. Dec 1880
Box Channel columns.
Lattice Channel columns


Breaking loads in lbs per square inch.
Breaking loads in pounds per square inch.
Values of length divided by least radius of gyration.
Notation.
P. Phoenix Iron Co. columns. Phoenixville U.S. tested at Watertown U.S
Trans. Am. Soc. Civ. Engrs. Feb. 1882.
W. Experiments Watertown Arsenal U.S. Report of 1881.
C. Experiments Cincinnati Southern Railway under the Chicago and Pittsburgh U.S. A. Boss curves C.E.
Trans. Am. Soc. C.E. Dec 1880
□ Box Channel columns.
× Lattice Channel columns
⊙ Phoenix Iron Co columns
I. American Bridge Co columns
Experiments by Mr James Christie C.E. Pencoyd Iron Works Pennsylvania U.S. Trans. Am. Soc. C.E. Apr. 1884
△ Flat Ends
⊼ Fixed Ends
L = Angles
T = Teas.
C = Channels
I = Beams.
WC = Watertown Channels
○ = Welded tubes.
Tie plates at ends
Lattice
Phoenix Bridge Co
Keystone Bridge Co
Z Post.
American Br. Co.
Channels with plates
Channels with lattice sides same form
Formula 46000 − 125 l/r